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# STRUCTURAL BEHAVIOUR OF HARDWOOD VENEER-BASED CIRCULAR HOLLOW SECTIONS OF DIFFERENT COMPACTNESS

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**Abstract:** This paper presents the capacity and structural behaviour of hardwood veneer-based circular hollow sections (CHS) tested in bending, shear and compression. The sections were manufactured from early to mid-rotation (juvenile) Gympie messmate (*Eucalyptus cloeziana*) plantation thinned logs. In total twenty-one 167 mm Outside Diameter (OD)  $\times$  1.2 m long CHS were manufactured in seven sets of three nominally identical sections. Two different wall thicknesses were investigated to produce nine compact and twelve more slender cross-sections. The sections were also manufactured in three different structural grades. A sudden failure mode was observed in the compression zone of the slender sections tested in bending. In compression, the compact sections showed a ductile behaviour, while the slender sections showed a more brittle behaviour, with the sections bursting into longitudinal strips. While a relationship was observed between the bending and compressive capacities, and the structural grade, no such relationship was noticed for the shear capacity. Comparison to steel and concrete sections of similar outside diameter proved that the timber sections are the most efficient in terms of bending and compressive capacity to linear weight ratio. The timber sections fall behind their steel and concrete counterparts in terms of shear efficiency, however they still have enough shear capacity for representative structural applications.

## 26 1. INTRODUCTION

27 To develop a market for low-value, small diameter, early to mid-rotation (juvenile) hardwood  
28 plantation logs, veneer-based hollow sections are currently being developed in Australia [1-3], see  
29 Figure 1. These sections have the potential to be used in structural applications [1, 3] and are seen,  
30 for instance, as a potential solution for utility poles [1] and the main frame of buildings. They have  
31 the advantage of having an efficient cross-sectional shape, are sustainable [4-6], and able to be man-  
32 ufactured in usable lengths [2] and cross-sectional sizes that are no longer available in sawn timber.

33 In the literature, various hollow timber structural solutions have been investigated. They include  
34 (i) spirally winded veneer-based Circular Hollow Section (CHS) [7-9], (ii) fibre-reinforced moulded  
35 wooden tubes [10-14], (iii) octagonal tubes from composite wood flakes panels [15], (iv) nonagon  
36 tubes from knot free pine wood strips [16], (v) “wood rings” reinforced with glass epoxy [17] and  
37 (vi) LVL type CHS for temporary geotechnical soil nailing systems [18]. Commercially, veneer-  
38 based hollow timber solutions are also available, either limited to small diameter cross-sections (up  
39 100 mm) [19] or short lengths (up to 1,000 mm) [20].

40 To confidently use the new sections in structural applications, research is still needed to fully un-  
41 derstand their structural behaviour, failure modes and reliability. In particular, bending tests per-  
42 formed on 145 mm Outside Diameter (OD)  $\times$  15 mm (wall thickness) Laminated Veneer Lumber  
43 (LVL) type CHS showed that the sections can experience a sudden failure in the compression zone,  
44 with the sections opening up [1]. While this failure mode has been observed in hollow trees [21], it  
45 is not typical of solid timber beams which usually reach a maximum bending moment due to tensile  
46 rupture [22]. The sudden compressive failure mode is likely attributed to the semi-compactness of  
47 the cross-section in [1] which led to local buckling and cross-section ovalisation (Brazier effect [23]).  
48 The relationship between the cross-sectional slenderness and structural behaviour requires further  
49 attention.

Consequently, the structural behaviour and failure modes of veneer-based timber CHS of various cross-sectional slenderness are experimentally investigated in bending, shear and compression in this paper. In total twelve 167 mm (OD)  $\times$  12.5 mm (wall thickness), referred to as “slender”, and nine 167 mm (OD)  $\times$  25 mm (wall thickness), referred to as “compact”, 1.2 m long CHS were manufactured from early to mid-rotation (juvenile) Gympie messmate (*Eucalyptus cloeziana*) plantation thinned logs. The veneer grain was orientated in the same direction and along the member longitudinal axis for all sections except for one type of the slender sections. For this section, cross-banded veneers were used in this case to potentially increase the section local buckling capacity. To study the effect of the timber elastic stiffness on the new products’ structural behaviour, the CHS were manufactured in three different structural grades. The grades were solely based on the veneers’ Modulus of Elasticity (MOE).

The paper initially introduces the investigated cross-sections and the associated manufacturing process. Secondly, the test set-ups for all investigated loading cases are presented. Thirdly, the structural behaviour, capacities and failure modes of the slender and compact sections are analysed and discussed. Finally, the performance of the studied sections is compared to similar steel and concrete counterparts.

## 2. INVESTIGATED CROSS-SECTIONS

### 2.1 General

In total, twenty-one nominal 167 mm (OD)  $\times$  1.2 m long veneer-based CHS were manufactured from two half cross-sections following the process described later in Section 2.2. Randomly selected nominal 1.2 m (Long)  $\times$  1.2 m (Wide)  $\times$  2.5 mm (Thick) Gympie messmate rotary peeled veneer sheets were delivered and then cut parallel to the grain direction (i.e. perpendicular to the length of the veneer ribbon) into four 300 mm wide strips. The longitudinal dynamic MOE of each veneer sheet

74 was then measured using a non-destructive resonance method [24]. To do so, the second cut strip per  
75 veneer sheet was simply supported on rubber bands and impacted with a hammer in its longitudinal  
76 direction. The sample natural frequency was recorded using a microphone and analysed using the  
77 software BING<sup>®</sup> (Beam Identification by Non-destructive Grading) [25]. Figure 2 shows a photo of  
78 the set-up. Before assessing the dynamic MOE, the veneers were conditioned in a temperature con-  
79 trolled room set at 22°C.

80 Based on their measured MOE, the delivered veneer sheets were divided into three stacks of equal  
81 number of veneers. This classified the veneers into three grades referred to as “Grade 1” for the lower  
82 MOE ( $13 \text{ GPa} < \text{MOE} \leq 19 \text{ GPa}$ ), “Grade 2” for the intermediate MOE ( $19 \text{ GPa} < \text{MOE} \leq 21 \text{ GPa}$ )  
83 and “Grade 3” for the higher MOE ( $21 \text{ GPa} < \text{MOE} \leq 25 \text{ GPa}$ ).

84 The twenty-one CHS were manufactured in seven sets of three nominal identical samples. Per set,  
85 the half cross-sections of the three nominally identical CHS were manufactured from the same veneer  
86 sheets which were glued in the exact same order. Precisely, for each veneer sheet, three 300 mm wide  
87 strips out of four were used in the CHS manufacturing process. The remaining strip was used to  
88 determine the material properties of the half cross-sections as detailed in Sections 2.2 and 3.2. The  
89 seven sets consisted of:

- 90 • Three sets of nominal 167 mm (OD)  $\times$  12.5 mm (5-ply) slender CHS manufactured from Grade 1  
91 (Set “S\_G1”), Grade 2 (Set “S\_G2”) and Grade 3 (Set “S\_G3”) veneers. In these sets, the veneers’  
92 grain is orientated in the same direction and along the longitudinal axis of the section.
- 93 • One set of nominal 167 mm (OD)  $\times$  13 mm slender CHS. To potentially increase the section local  
94 buckling capacity, a cross-banded configuration was used. Four 2.5 mm thick Gympie messmate  
95 hardwood Grade 2 veneers were orientated along the longitudinal axis of the section and three 1  
96 mm thick cross-banded softwood Hoop pine (*Araucaria cunninghamii*) veneers were inserted be-  
97 tween the hardwood veneers to form a 7-ply configuration. This set is referred to as “S\_G2\_Cross”.

- Three sets of nominal 167 mm (OD)  $\times$  25 mm (10-ply) compact CHS manufactured from Grade 1 (Set “C\_G1”), Grade 2 (Set “C\_G2”) and Grade 3 (Set “C\_G3”) veneers. In these sets, the veneers’ grain is orientated in the same direction and along the longitudinal axis of the section.

An examples of a compact and slender CHS is shown in Figure 1 (a).

Note that while the wall the slender sections is quite thin, fire protection may be achieved by gluing sacrificial low MOE veneers to the outside of the sections, therefore protecting the load carrying part of the CHS.

## **2.2 Manufacturing process**

The manufacturing process detailed in [18, 26] and used to manufacture the samples tested in [1, 2] has been improved in this study. A similar process to the one described in [18, 27] has been followed. After assessing the dynamic MOE of the veneers, the veneers were moved out of the temperature controlled room and stored in an indoor environment (structure laboratory) until gluing. To form the half cross-sections, resorcinol formaldehyde structural adhesive was applied to the veneer strips at ambient temperature and humidity. The veneer stacks were then inserted into a 167 mm Internal Diameter (ID) CHS PVC pipe and cold-pressed for 24 hours by a fire hose inserted into the PVC pipe and pressurised at 1.2 MPa with water. Figure 3 illustrates the manufacturing process.

As rotary peeled veneers have the natural tendency to curl about their loose side (i.e. the one in contact with the blade of the peeling lathe), the loose side of a veneer was always glued herein to the tight side of the next veneer. The tight and loose veneer sides therefore formed the outside and inner faces of the manufactured hollow cross-sections, respectively. The two half cross-sections forming a complete CHS were then butt jointed together using structural epoxy resin (Figure 1 (b)) due to its good gap properties which can compensate for non-strict parallelism of the two half cross-sections. For alignment, the glue-line incorporated biscuit joints every 400 mm.

121        Additionally, to determine the mechanical properties of the material of the timber sections, two  
122        500 mm × 300 mm flat panels were also manufactured for each half-section. The panels were manu-  
123        factured from the same veneer sheets used to produce the half cross-sections and were glued in the  
124        exact same layering order.

125

### 126    **3. TESTING METHODOLOGY**

#### 127    **3.1    *General***

128        Per manufactured set, one section was tested in bending, one in shear and one in compression. The  
129        following sub-sections introduce the material testing methodology and the test set-ups of the CHS for  
130        each one of the investigated loading cases.

131        Before testing, all samples were conditioned in the same temperature controlled room as the ve-  
132        neers when the dynamic MOE was assessed, for a minimum period of one month. The temperature  
133        in the room was set at 22°C. For all scenarios, excluding the CHS tested in compression, pieces were  
134        cut and weighed immediately after testing from selected test samples to determine the timber moisture  
135        content at the time of testing. The oven-dry methodology in the Australian and New Zealand standard  
136        AS/NZS 1080.1 [28] was followed.

#### 137    **3.2    *Material properties***

##### 138    **3.2.1    *Tension tests***

139        From the first flat panel of each half cross-section, a maximum of five nominal 10 mm wide × 100  
140        mm long (gauge length) coupon (dog bone) samples were CNC cut. The samples were similar to the  
141        ones recommended by the ASTM D3500–14 [29] and were used to estimate the tensile strength of  
142        each half cross-section. The ends of the samples were clamped in the jaws of a 500 kN capacity MTS  
143        universal testing machine and tested in tension at a constant strain rate to reach failure in 3-6 mins.

144        The tensile strength  $\sigma_{tens}$  of each coupon was calculated as,

145

$$\sigma_{tens} = \frac{F_{max}}{W_t t_t} \quad (1)$$

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where  $F_{max}$  is the maximum recorded force,  $W_t$  and  $t_t$  are the measured width and thickness of the coupons, respectively.

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### 3.2.2 Compression tests

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The second flat panel of each half cross-section was used to determine the compressive strength of the material. To avoid buckling of the samples corresponding to the slender CHS, the 12.5 mm thick panels were cut in two and glued together using resorcinol formaldehyde structural adhesive to form nominal 25 mm thick panels. The panels corresponding to the compact CHS were left untouched. Up to four 80 mm (Wide)  $\times$  150 mm (Long) rectangular samples were cut per panel for material testing.

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The samples were tested in compression in a 500 kN capacity MTS universal testing machine at a constant strain rate to reach the peak stress in 3-5 mins. Specifically, the samples were positioned between a fixed bottom platen and an upper platen mounted on a spherical seat, which could rotate, so as to provide full contact between the platens and the specimens. Note that before testing, the ends of the samples were cut with a high quality fine cut circular saw blade to ensure a uniform contact pressure between the platens and the samples.

161

Similar to Eq. (1), the compressive strength  $\sigma_{comp}$  of each sample was calculated as,

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$$\sigma_{comp} = \frac{F_{max}}{W_c t_c} \quad (2)$$

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165

where  $F_{max}$  is the maximum recorded force,  $W_c$  and  $t_c$  are the measured width and thickness of the samples, respectively.



### 166 3.3 *Bending tests*

#### 167 3.3.1 *Test set-up*

168 To measure the bending strength and stiffness of the timber CHS, the sections were tested in a  
169 similar manner to the one reported in [2]. A pair of four reinforced quarter steel tubes, 240 mm long,  
170 were designed and manufactured to rigidly clamp each end of the CHS, as shown in Figure 4. Each  
171 steel clamp was bolted to a steel Rectangular Hollow Section (RHS) to form a 2,360 mm long beam.  
172 To avoid local crushing of the timber CHS and fully transfer the moment from the steel RHS to the  
173 timber with minimum stress concentration, two part epoxy resin was poured at the steel-timber con-  
174 nection (i) in the inside of the timber CHS filled with plywood and (ii) on the outside of the timber  
175 CHS to match the inside diameter of the four quarter steel tubes. On top of the friction forces applied  
176 by the clamps to the timber, screws connecting the steel to the timber were also added to further  
177 prevent sliding of the timber sections in the clamps. The overall test set-up is shown in Figure 5.

178 The sections were then tested in a 500 kN capacity MTS universal testing machine, with the load  
179 being applied to the steel RHS, as shown in Figure 5. The tests were run in displacement control and  
180 reached failure in 3-4 minutes for the slender sections and 5-6 minutes for the compact sections. For  
181 all tests, the butt joints between two half-sections lied in the horizontal plane.

182 Three Laser Displacement Sensors (LDS) recorded the vertical displacement at the bottom fibre  
183 of the timber sections for simplicity in the test set-up. Additionally, two 30 mm strain gauges (SG)  
184 recorded the mid-span longitudinal strain at the top (compression) and bottom (tension) fibres of the  
185 timber CHS. A third 30 mm strain gauge recorded the mid-span tangential stress to better apprehend  
186 the cross-sectional deformation. Locations and numbering of all LDS and strain gauges are given in  
187 Figure 5 (b). The 300 mm distance between LDS was chosen so the edge LDS are away for the  
188 clamping ends while placing the LDS the further away from each other.

#### 189 3.3.2 *Evaluations*

190 The applied moment  $M$  to the hollow timber sections is calculated as,

$$M = \frac{(F + F_w)L_1}{2} \quad (3)$$

where  $F$  is the total applied load,  $F_w = 2.37$  kN is the gravity load applied by the steel rig (including the steel CHS and measured at the points of application of the load) and  $L_1 = 455$  mm is given in Figure 5 (b). The bending capacity  $M_b$  is defined as the maximum applied moment  $M$  and the bending strength  $f_b$  is obtained from the well-known equation,

$$f_b = \frac{M_b}{Z} \quad (4)$$

where  $Z$  is the section modulus calculated from the measured cross-sectional dimensions, assuming a perfect composite action between the two half cross-sections.

The relative displacement  $\delta$  of the timber sections is calculated from the displacements  $\delta_1$ ,  $\delta_2$  and  $\delta_3$  recorded by the LDS number 1, 2 and 3, respectively, as,

$$\delta = \delta_1 - \frac{\delta_2 + \delta_3}{2} \quad (5)$$

The static MOE  $E_s$  parallel to the grain of the timber sections is calculated from the bending stiffness  $E_s I_s$  defined as,

$$E_s I_s = \frac{k_t d^2}{2} \quad (6)$$

where  $I_s$  is the second moment of area of the CHS (calculated from measured dimensions),  $d$  is given in Figure 5 (b) and  $k_t$  is the stiffness of the linear part of the experimental moment-displacement curve ( $M$ - $\delta$ ), calculated by performing a linear regression between 5 kN.m and 20 kN.m for the compact sections and 2.5 kN.m and 15 kN.m for the slender ones. Note that Eq. (6) assumes that the relative displacement  $\delta$  is measured at the neutral axis. Yet, using the relative displacement measured in this study at the bottom fibre of the section provides accurate results, with a maximum error in determining  $E_s I_s$  of less than 0.5%.

### 212 3.4 Shear tests

#### 213 3.4.1 Test set-up

214 To estimate the shear strength of the timber CHS, the sections were tested in three point bending,  
215 similarly to the tests performed in [1]. The sections were simply supported with a distance  $L = 500$   
216 mm between two consecutive loads, as shown in the schematic test set-up in Figure 6. To avoid local  
217 crushing of the sections, two part epoxy resin (combined with plywood) was poured inside the CHS  
218 at the load application point and supports. The butt joints between two half cross-sections lied in the  
219 horizontal plane. For each set, the half cross-section which was in compression in the bending test  
220 (Section 3.3) was also in compression in the shear test. The tests were performed in a 500 kN capacity  
221 MTS universal testing machine in displacement control and reached failure in 6-8 minutes for all  
222 sections but for S\_G3 which was tested at a higher strain rate and reached failure in 2 minutes.

#### 223 3.4.2 Evaluations

224 The shear strength  $f_s$  of the hollow timber sections is calculated using the shear area of a CHS as  
225 [30],

$$226 \quad f_s = \frac{F_{\max}}{\frac{3}{2} A \left( \frac{R_o^2 + R_i^2}{R_o^2 + R_o R_i + R_i^2} \right)} \quad (7)$$

227 where  $F_{\max}$  is the total maximum applied load,  $A$  is the measured CHS cross-sectional area, and  $R_o$   
228 and  $R_i$  are the measured CHS external and internal radii, respectively. The shear capacity  $V_s$  is calcu-  
229 lated as  $F_{\max}/2$ .

### 230 3.5 Compressive tests

#### 231 3.5.1 Test set-up

232 To measure the compressive strength and stiffness of the timber CHS, the sections were tested in  
233 compression in a 10 MN capacity MTS universal testing machine. The sections were positioned be-  
234 tween a fixed bottom platen and an upper platen mounted on a spherical seat, which could rotate. The

235 samples were mechanically sanded flat in a milling machine before testing to ensure a uniform contact  
236 pressure between the platens and the CHS. The tests were performed in displacement control and  
237 reached failure in 3-4 minutes for the slender sections and 5-6 minutes for the compact sections. The  
238 test set-up is shown in Figure 7.

239 Two diametrically opposed 30 mm strain gauges, glued parallel to the column axis, each located  
240 in the middle of a half cross-section and 150 mm from the bottom end of the sections, recorded the  
241 longitudinal deformation. Strain gauges numbering is given in Figure 7.

#### 242 3.5.2 Evaluations

243 The compressive stress  $\sigma$  of the hollow timber sections is calculated as,

$$244 \quad \sigma = \frac{F}{A} \quad (8)$$

245 where  $F$  is the applied load and  $A$  is the measured CHS cross-sectional area. The compressive capacity  
246  $R_c$  and strength  $f_c$  are defined as the maximum applied force and compressive stress, respectively.  
247 The static MOE  $E_s$  is calculated by performing a linear regression on the linear part of the stress-  
248 strain curve ( $\sigma$ - $\varepsilon$ ) between 5 MPa and 40 MPa. The strain  $\varepsilon$  is calculated as the average of strains  $\varepsilon_1$   
249 and  $\varepsilon_2$  from strain gauges 1 and 2, respectively.

250

## 251 4. RESULTS AND DISCUSSION

### 252 4.1 Material properties

253 Table 1 gives the tensile and compressive strengths of the material of each half cross-section of  
254 each investigated set. As the veneer MOE increases with the grade, so typically does the measured  
255 material strength [31]. For the LVL samples, the compressive strength ranges from 58.6 MPa (S\_G1)  
256 to 77.9 MPa (C\_G3 and S\_G3), and the tensile one from 96.3 MPa (C\_G1) to 135.8 (C\_G3). Due to  
257 the nature of the brittle tensile failure mode compared to the ductile compressive failure mode of

timber samples, the Coefficients of Variation (CoV) of the tensile test results are typically higher than the ones of the compressive test results. The average oven dry moisture content at the time of testing of the tension and compression samples is reported in Table 2.

## **4.2 Bending tests**

### **4.2.1 Capacities and failure modes**

The bending capacities  $M_b$  and strengths  $f_b$  for all CHS tested in bending are reported in Table 3, along with the measured static MOE  $E_s$  (Eq. (6)) and observed failure modes. Two of the slender sections (S\_G1 and S\_G3) failed in buckling of the compression fibre, with the section opening up, as shown in Figure 8 (a). Slender S\_G2 and compact C\_G1 sections failed in tensile rupture, as shown in Figure 8 (b). The cross-banded CHS (C\_G2\_Cross) prematurely failed in the butt joint between the two half cross-sections, as shown in Figure 8 (c). This weak zone was only observed for all testing configurations in C\_G2\_Cross, as later reported in Sections 3.4 and 0. In all other sections and sections tested in [1, 2], failure never developed in the butt joint. For compact sections C\_G2 and C\_G3, the steel clamps did not provide sufficient restraints and the sections ultimately slid at the steel-timber connections, leading to shear failure, as shown in Figure 8 (d). However, the maximum bending stresses reached for these two sections are higher than the bending strengths  $f_b$  of all other tested sections. It is therefore very likely that the maximum recorded moments are within a few percent of the bending capacities  $M_b$  of the sections. Noting that these maximum recorded moments represent lower bound values of  $M_b$ , their values are conservatively taken for  $M_b$  herein for both C\_G2 and C\_G3 sections. For all sections, the bending strength typically increases with the veneer MOE (or grade).

The compact sections reached on average a bending strength  $f_b$  18% higher than the one of the slender sections of the same grade. This result is attributed to different material strengths between sections (Table 1) and possibly to the section compactness. Indeed, when buckling develops in the compression zone of the slender sections, it would result in a loss in stiffness of the section wall,

consequently inducing a shift of the neutral axis and a higher stress in the tension zone. The sections would eventually fail in the compression zone (S\_G1 and SG\_G3) or tension zone (S\_G2), whichever zone is the weakest. Such phenomenon would not occur for compact sections for which the compressive zone only experiences plasticity without buckling, as typically observed in timber beams [22]. A similar tensile failure mode to the one experienced in timber beams would be therefore expected.

More investigations are needed to (i) fully comprehend the mechanisms involved in the observed failure modes of the slender sections, (ii) validate the hypothesis in the above paragraph and (iii) quantify the influence of the cross-sectional geometry, timber compressive and tensile strengths on the full section capacity. Numerical models, similar to the one developed in [1], can be used to predict the capacity of compact sections.

Note that the cross-banded section (S\_G2\_Cross) has a bending strength  $f_b$  and static MOE  $E_s$  9% and 55%, respectively, lower than the ones of the slender section of the same grade (S\_G2). Cross-banded veneer-based CHS would gain further structural optimisation, such as number and thickness of the cross bands.

#### 4.2.2 Behaviour

Figure 9 plots the Moment-Displacement curves ( $M-\delta$ ) of all investigated sections. While a large non-linear behaviour is observed for the compact sections, it is limited for the slender sections, except for the cross-banded one. As outlined in Figure 9, when failure occurred the moment suddenly dropped for all sections. This observed drop for the two slender sections failing in buckling of the compression zone (S\_G1 and S\_G3) is due to the sections opening up.

Figure 10 (a) shows the readings of the two strain gauges glued in the section longitudinal axis (SG 1 and SG 3). Timber elements loaded in tension typically exhibit a linear behaviour until fracture suddenly occurs at the maximum tensile strength, and the strain recorded on the tension zone (SG 3)

306 is consequently almost linear. Plasticity occurred on the compression side (SG 1) at an applied mo-  
307 ment of about 20-25 kN.m and 12-15 kN.m for the compact and slender sections, respectively. This  
308 corresponds to bending stresses of about 60 MPa to 75 MPa, i.e. of the same order of magnitude of  
309 the material compressive strengths reported in Table 1. Due to the buckling of the compression zone  
310 for S\_G1 and S\_G3, reading of Strain gauge 2 reached a plateau for these sections at about 12,000 to  
311 16,000  $\mu\epsilon$ .

312 The transverse strain recorded by SG 2 is plotted in Figure 10 (b). The figure mainly indicates that  
313 the transverse strain significantly increased when plasticity damage occurred in the compression zone.  
314 The strain reversal experienced for S\_G1 and S\_G3 is likely attributed to the buckling and ovalisation  
315 of the cross-sections.

316 The average oven dry moisture content at the time of testing of the sections tested in bending is  
317 reported in Table 2.

### 318 **4.3 Shear tests**

319 Table 4 gives the experimental shear capacities  $V_s$  and strengths  $f_s$  for all investigated sections. All  
320 sections failed in the timber except S\_G2\_Cross which failed in the butt joint between the two half  
321 cross-sections. The two observed failure modes are shown in Figure 11. All sections reached a similar  
322 shear strength of 10 MPa, +/- 7%, indicating that contrary to the bending tests, the grade does not  
323 influence the shear capacity. Note that despite S\_G2\_Cross failing in the butt joint, it still reached a  
324 strength of 10.4 MPa. Further optimisation of the cross bands layering may improve the shear capacity  
325 of the CHS.

326 In terms of shear capacities, the slender sections sustained shear forces up to 32 kN and the com-  
327 pact ones up to 60.8 kN.

328 The average oven dry moisture content at the time of testing of the sections tested in shear is  
329 reported in Table 2.

#### 330 4.4 Compressive tests

331 The compressive stress-strain curves ( $\sigma$ - $\epsilon$ ) of all sections are plotted in Figure 12. Two different  
332 types of curves are observed resulting in two different failure modes. The compact sections showed  
333 a large non-linear plastic behaviour, with the load reaching a plateau before gradually decreasing. A  
334 portion of the section wall ultimately locally buckled in a compression type failure mode, as shown  
335 in Figure 13 (a). This led to a sudden drop of the load with the section remaining in one single piece.  
336 For the slender sections, a plastic behaviour usually started to develop but premature failure (i.e.  
337 before the load reached a plateau as for the compact sections) suddenly occurred with the sections  
338 bursting into (i) two half cross-sections, with the failure occurring in the butt joint, for S\_G2\_Cross  
339 and (ii) six to seven strips for all remaining slender sections. The latter failure mode is shown in  
340 Figure 13 (b) and was also observed in [13, 32] for formed wood profiles. The slender sections could  
341 therefore not reach their potential full capacity and exhibited a failure mode which should be avoided  
342 in structures.

343 The compressive capacities  $R_c$  and strength  $f_c$  for all tested sections are reported in Table 5. The  
344 ratios of  $f_c$  to the average material compressive strength  $\sigma_{comp}$  of the two half cross-sections (reported  
345 in Table 1) are also given in the table. Similar to the bending tests, the compressive strength increases  
346 with the veneer MOE (grade). Interestingly, both slender and compact sections reached a capacity  
347 higher, up to 20%, than the one of the average measured compressive strength of the material. This  
348 observation is in contradiction with the length effect [33, 34] encountered in timber structures for  
349 which the larger the tested volume, the lesser the capacity. The circular shape of the section may  
350 delay the compression failure of the cell walls when compared to the results reported in Section 4.1  
351 and performed on flat panels. Further investigations are needed to validate and understand the ob-  
352 served phenomena.



353 The compressive capacity is high for both section types and reached about 400-500 kN for the  
354 slender sections to about 800-1,000 kN for the compact ones. For the compact sections, the capacity  
355 is in the range of the design load which may be encountered in the columns of mid-rise timber build-  
356 ings.

357 Also given in Table 5 are the static MOE  $E_s$  of the sections measured from the linear part of the  
358 stress-strain curves ( $\sigma$ - $\epsilon$ ) and the ratios of  $E_s$  measured from the bending tests to the one measured  
359 from the compressive tests. The values of  $E_s$  measured from the two types of tests are consistent with  
360 an average difference between the two values of 4%.

## 361 **4.5 Comparisons**

362 The structural efficiency of the compact sections is compared herein to the one of steel and rein-  
363 forced concrete circular sections of similar (i) diameter and (ii) compressive short-term capacity to  
364 the middle grade section C\_G2 reported in Table 5. In a first instance only the short-term capacities  
365 of the timber section is compared to the ultimate capacities of the steel and concrete counterparts,  
366 which are calculated based on relevant Australian standards and without the use of the capacity factor  
367 (resistance factor). Effect on long-term loading on the structural efficiency is discussed in a second  
368 instance.

### 369 **4.5.1 Comparison to steel CHS**

370 A 168.3 (OD)  $\times$  4.8 (wall thickness) CHS, commercialised by the Australian manufacturer  
371 Onesteel [35], is selected for the structural steel section. Its yield stress is 350 MPa. Based on the  
372 Australian and New-Zeeland standard AS4100 [36], the steel section has ultimate bending, shear and  
373 compressive section capacities of  $M_b = 44.8$  kN.m,  $V_s = 311.2$  kN and  $R_c = 864.5$  kN, respectively.  
374 Its compressive capacity is within 4% of the one of C\_G2.

375 Table 6 compares the ultimate capacities, ultimate capacity to linear weight ratios, bending and  
376 axial stiffness of the steel and timber sections. Densities of  $805 \text{ kg/m}^3$  for early to mid-rotation Gym-  
377 pie messmate veneers [37] and  $7,850 \text{ kg/m}^3$  for steel are used in Table 6. MOE of 200 GPa is also  
378 used for the steel in the Table.

379 Results show that the timber CHS has a short-term bending capacity  $M_b$  comparable to and only  
380 13% lower than the one of the steel CHS. Yet, the timber CHS is nearly twice more efficient in terms  
381 of ultimate capacity to linear weight ratio. A similar conclusion applies to the compressive capacity  
382 to linear weight ratio, with the timber section being more than twice more efficient than the steel  
383 CHS. However, the steel CHS is stiffer than the timber profile, with the bending and axial stiffness  
384 being 2.7 and 2.1 times higher, respectively.

385 Regarding the shear, the timber CHS performs poorly when compared to the steel profile. The  
386 shear capacity  $V_s$  and shear to linear weight ratio of the steel CHS are 5.1 and 2.4 times higher, re-  
387 spectively, than the ones of the timber sections. However, for the sizes of timber beams typically  
388 encountered in structural applications, i.e. with a span to depth ratio of 20 [38], the shear capacity of  
389 the timber section would be high enough. A simply supported, 3.5 m long,  $167 \times 25$  timber CHS loaded  
390 with a UDL which fails at an ultimate bending moment of 39.1 kN.m (C\_G2 in Table 3), would  
391 experience a maximum shear force of 44.7 kN. This shear force is 26% lower than the shear capacity  
392 recorded for C\_G2 in Table 4.

393 For long-term loading, the Australian standard AS1720.1 [39] uses a duration of load factor of  
394 0.57. Therefore, using the same  $168.3 \times 4.8$  steel CHS and comparing it to the timber CHS, but under  
395 long-term loading, the timber section becomes 1.08 and 1.28 times more efficient than the steel CHS  
396 in term of bending capacity to linear weight ratio and compressive capacity to linear weight ratio,  
397 respectively. In terms of shear capacity to linear weight ratio, the timber CHS now becomes 4.16  
398 times less efficient than the steel CHS.

399 4.5.2 *Comparison to reinforced concrete plain circular section*

400 A 167 mm diameter plain reinforced concrete column, with a concrete compressive strength  $f'_c =$   
401 40 MPa and a steel yield stress  $f_y = 500$  MPa, was designed to standard practices and the Australian  
402 Standard AS3600 [40]. While it is understood that a 167 mm diameter concrete columns would usu-  
403 ally not be used in practice, it still forms a comparative solution to the performance of the timber  
404 section. The concrete section is shown in Figure 14, has four N12 longitudinal reinforcing bars and  
405 an R10 helix with a pitch of 150 mm. Based on the requirements in [40], a minimum concrete cover  
406 of 20 mm is used with a minimum of 2% reinforcing steel by gross cross-sectional area. The AS3600  
407 [40] gives ultimate bending, shear and compressive capacities of  $M_b = 11.9$  kN.m,  $V_s = 92$  kN and  $R_c$   
408  $= 946.4$  kN, respectively, for the concrete section. The compressive capacity of this column is there-  
409 fore within 6% of the one of C\_G2.

410 Table 6 compares the ultimate capacities, ultimate capacity to linear weight ratios, bending and  
411 axial stiffness of the concrete and timber sections. The density of concrete for the calculations pre-  
412 sented is  $2,400 \text{ kg/m}^3$  and the MOE is  $32,8 \text{ GPa}$ , in accordance to [40].

413 The ultimate bending capacity of the concrete section is significantly lower (3.3 times lower) than  
414 the short-term bending capacity of the timber section. This results in the timber section being 22 times  
415 more efficient than the concrete one in terms of bending capacity to linear weight ratio. On the other  
416 hand, the bending stiffness of the concrete section is twice higher than the proposed timber section  
417 and nearly as stiff as the steel section. Note that the small diameter of the concrete column results in  
418 the steel being placed close to the neutral axis and therefore an inefficiency in resisting bending mo-  
419 ments is introduced. It is anticipated that for columns of larger diameter, the efficiency of the concrete  
420 column for these comparisons would improve.

421 In terms of shear, the shear capacity of the reinforced concrete section is 1.5 higher than the short-  
422 term shear capacity of the timber section, yet the concrete solution is 3.9 less efficient than the timber  
423 one in terms of shear capacity to linear weight ratio.

424 The concrete section is also the least efficient option in terms of compressive capacity to linear  
425 weight ratio. It is 5.6 times and 2.5 less efficient than its timber and steel counterparts, respectively.  
426 Nevertheless, it outperformed both the steel (1.5 times higher) and timber (3 times higher) solutions  
427 in terms of compressive stiffness.

428 Regarding long-term loading and considering a duration of load factor of 0.57 [39] on the results  
429 in Table 6, the concrete section becomes 12.5, 2.2 and 3.2 times less efficient in terms of bending,  
430 shear and compressive capacity to linear weight ratios, respectively, when compared to the long-term  
431 loading capacities of the timber section.

432

## 433 5. CONCLUSION

434 This paper presented the bending, shear and compression capacities, and structural behaviour, of  
435 hardwood veneer-based CHS manufactured from early to mid-rotation (juvenile) Gympie messmate  
436 plantation thinned logs. Twelve 167 mm (OD)  $\times$  12.5 mm (wall thickness), referred to as “slender”,  
437 and nine 167 mm (OD)  $\times$  25 mm (wall thickness), referred to as “compact”, 1.2 m long CHS were  
438 produced in seven sets of three nominally identical sections. The sections were tested in bending,  
439 shear and compression. A sudden failure mode was observed in the compression zone of the slender  
440 sections tested in bending, while the compact sections failed in the tension zone. The section had  
441 shear capacities of the same order of magnitude, within 7% of each other. In compression, the com-  
442 pact sections showed a ductile behaviour, while the slender sections catastrophically failed, with the  
443 sections bursting into six to seven longitudinal strips. The section compressive strength was observed  
444 to be consistently higher than the compressive strength of the material determined from tests per-  
445 formed of flat samples. Comparison to steel and concrete sections of similar outside diameter proved  
446 that the timber sections were the most efficient in terms of bending and compressive capacity to linear  
447 weight ratio. However, while the timber sections fell behind their steel and concrete counterparts in

terms of shear efficiency, they still showed enough shear capacity for structural applications. The optimisation of the cross-banded layering may improve the shear capacity without significantly impacting the critical structural performances of the CHS.

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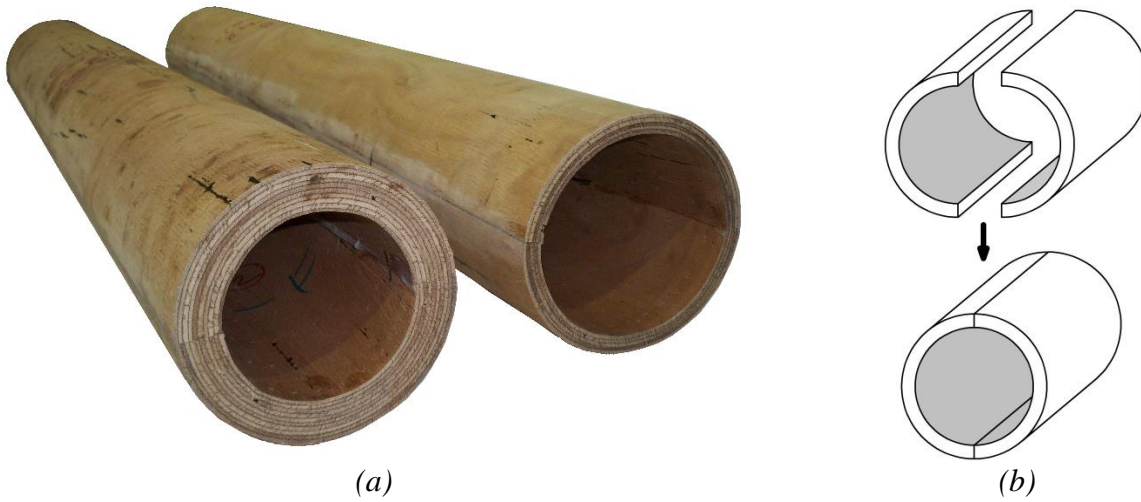
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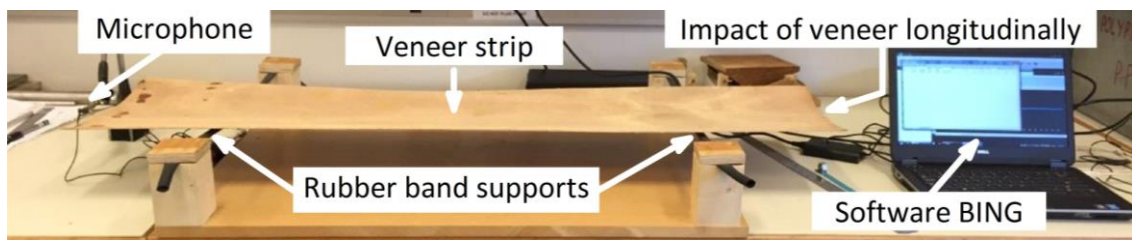
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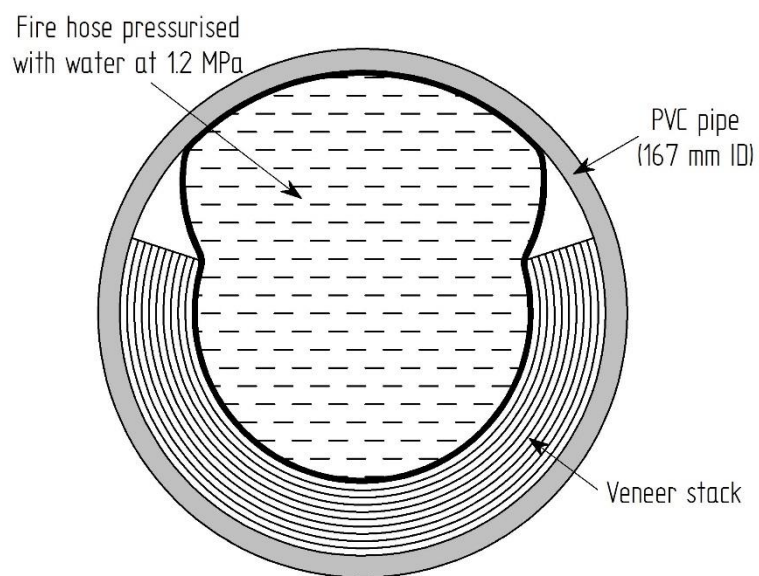




**Figure 1:** (a) circular hollow section currently developed in Australia (shown for compact and slender 167 mm (OD) Gympie messmate) and (b) principle of half cross-sections butt joined together to form a complete CHS

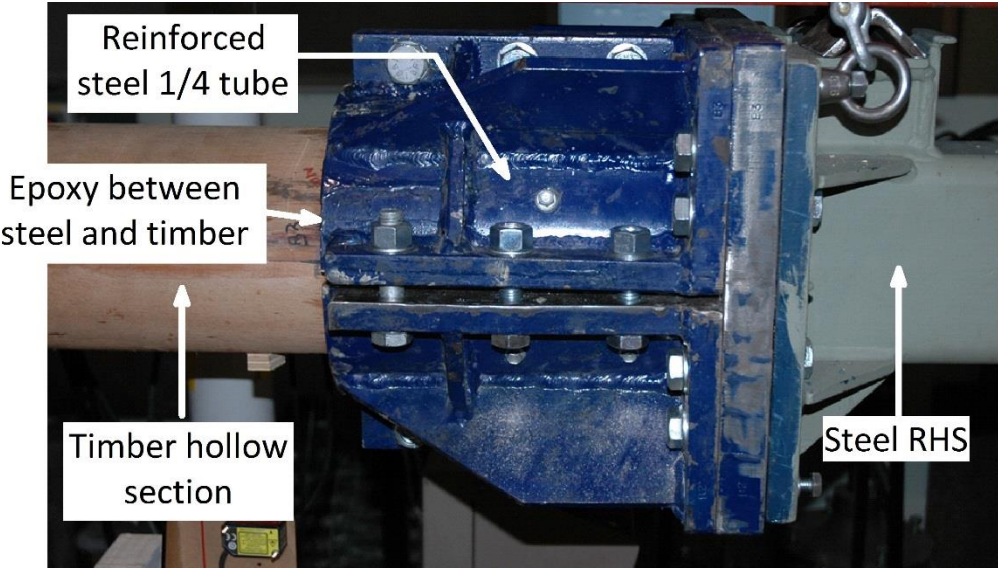


**Figure 2:** Set-up to assess the longitudinal MOE



**Figure 3:** Manufacturing process of the half cross-sections

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572

573

*Figure 4: Clamps to connect timber CHS to test rig*

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575

Roller support (slotted holes)

Roller

Strain gauge

MTS testing machine

Spreader bar

Pinned support

Knife edge

Steel RHS

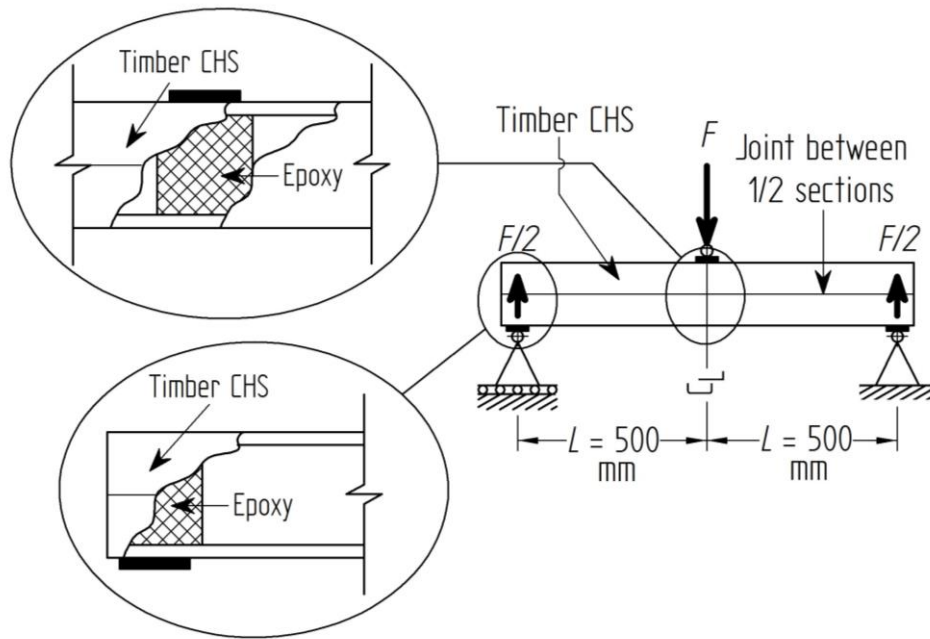
Laser Displacement Sensor

Timber CHS

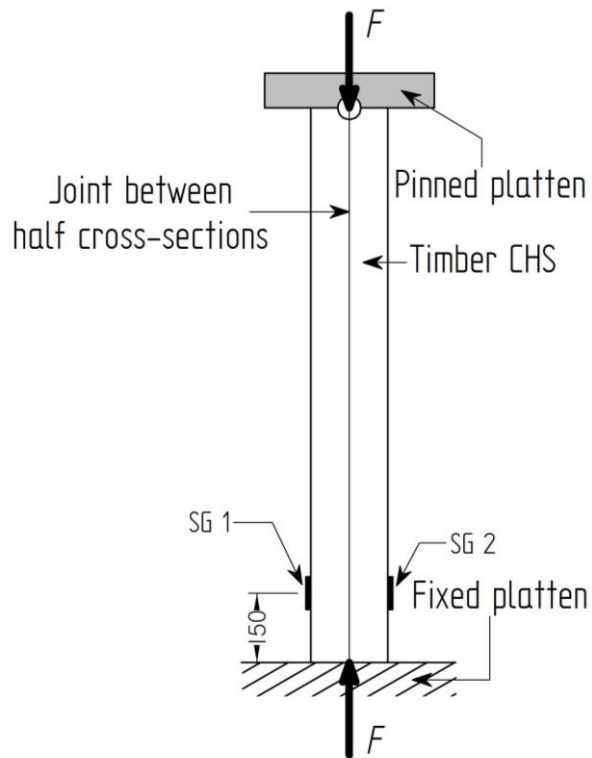
Steel RHS

The diagram illustrates the experimental setup for a timber-steel composite beam. The main schematic shows a beam of total length  $L = 2,360$  mm, divided into three sections:  $L_1 = 455$  mm (Steel RHS),  $L_2 = 1,450$  mm (Timber CHS), and  $L_1 = 455$  mm (Steel RHS). The beam is supported at both ends by a pin support and a roller support. A central vertical line marks the 'Joint between 1/2 sections'. Three strain gauges (SG 1, SG 2, SG 3) are located on the top flange of the Timber CHS section. A clamping device is used to connect the Steel RHS and Timber CHS sections. Two circular insets provide details: the left inset shows the clamping device with epoxy filling the gap between the Steel RHS and Timber CHS; the right inset shows the cross-section of the Timber CHS with SG 1, SG 2, and SG 3 located 33 mm from the top edge.

**Figure 5:** Bending test set-up, (a) overall picture and (b) schematic



**Figure 6: Shear test set-up**



**Figure 7: Compression test set-up**

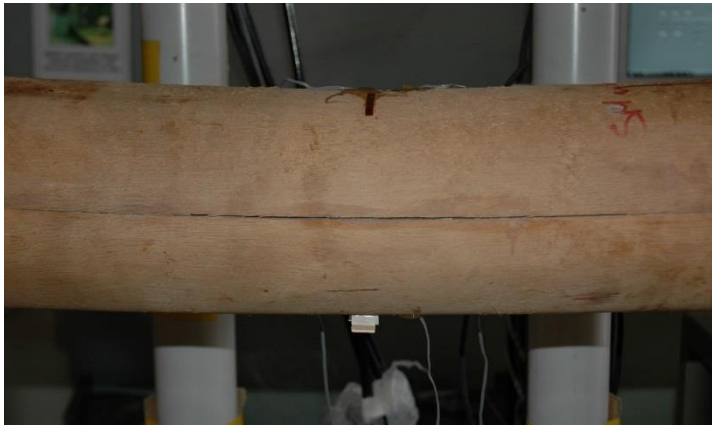




(a)



(b)

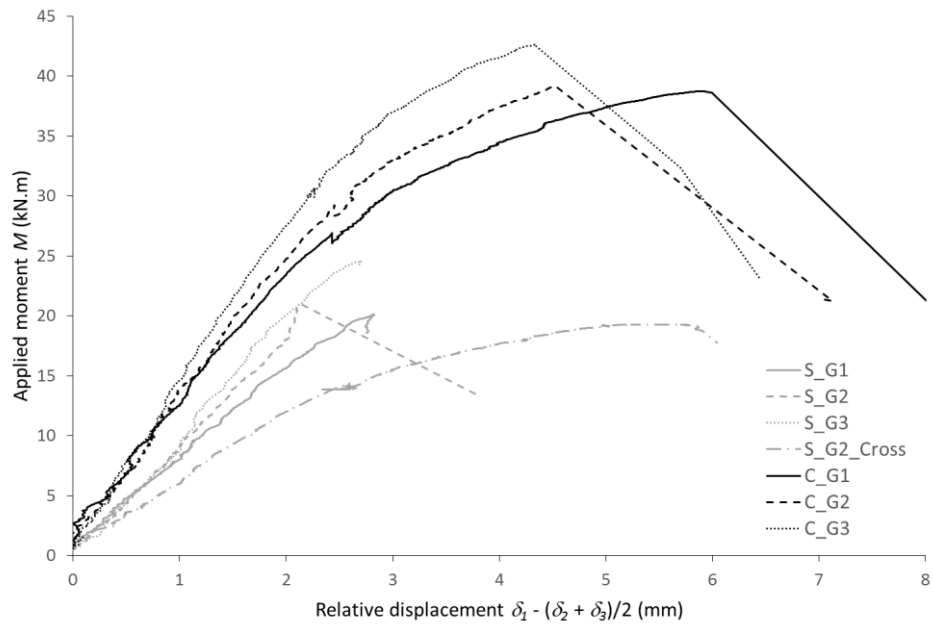


(c)



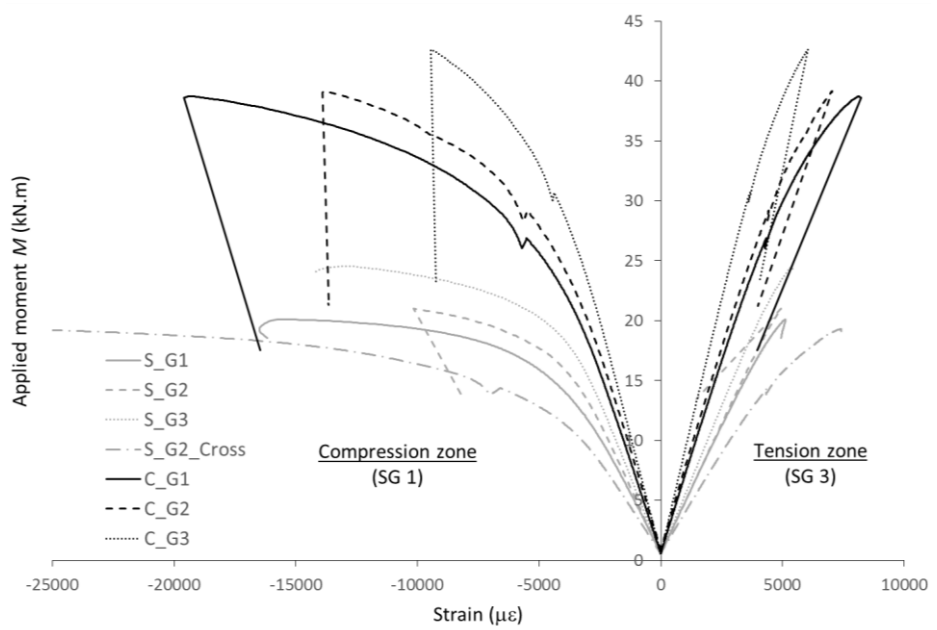
(d)

**Figure 8:** Bending tests failure modes (a) buckling of the compression zone (shown for S\_G3), (b) Tensile rupture (shown for C\_G1), (c) initial failure in the butt joint for C\_G2\_Cross and (d) premature failure at the steel-timber connections (shown for C\_G2)



**Figure 9:** Bending tests, Moment-Displacement curves ( $M$ - $\delta$ ) for all investigated sections

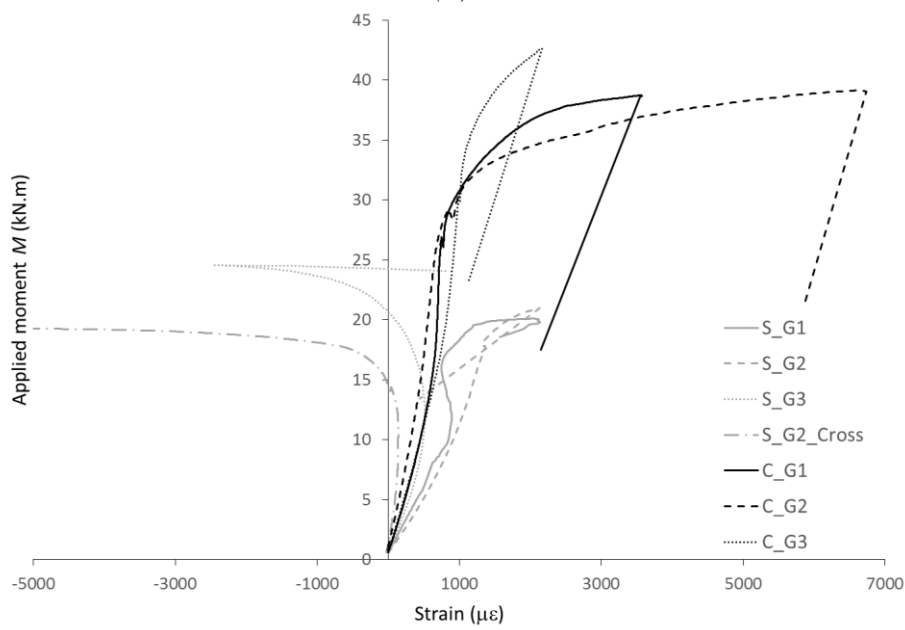
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(a)



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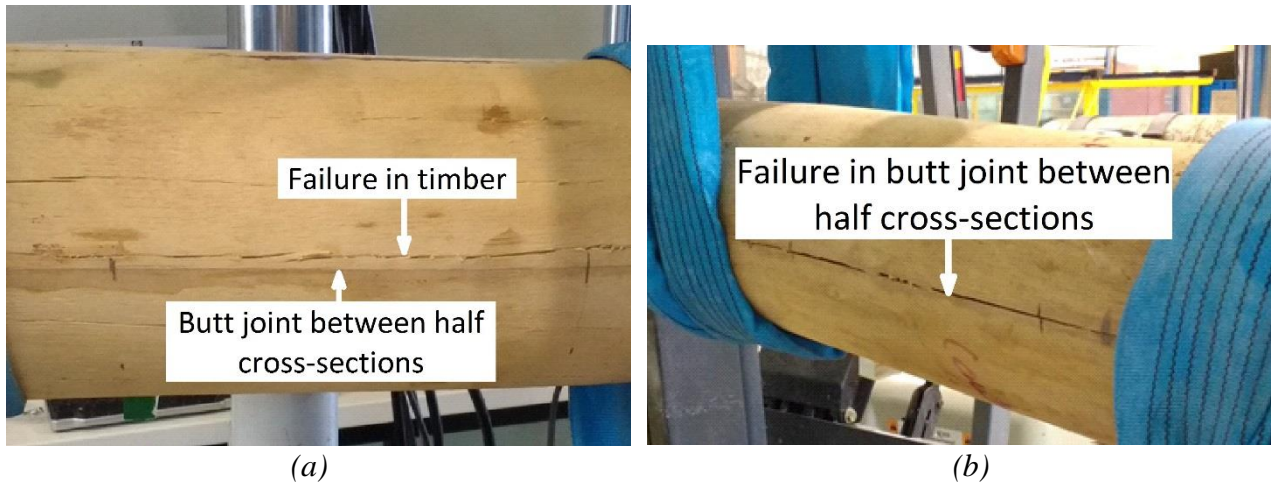
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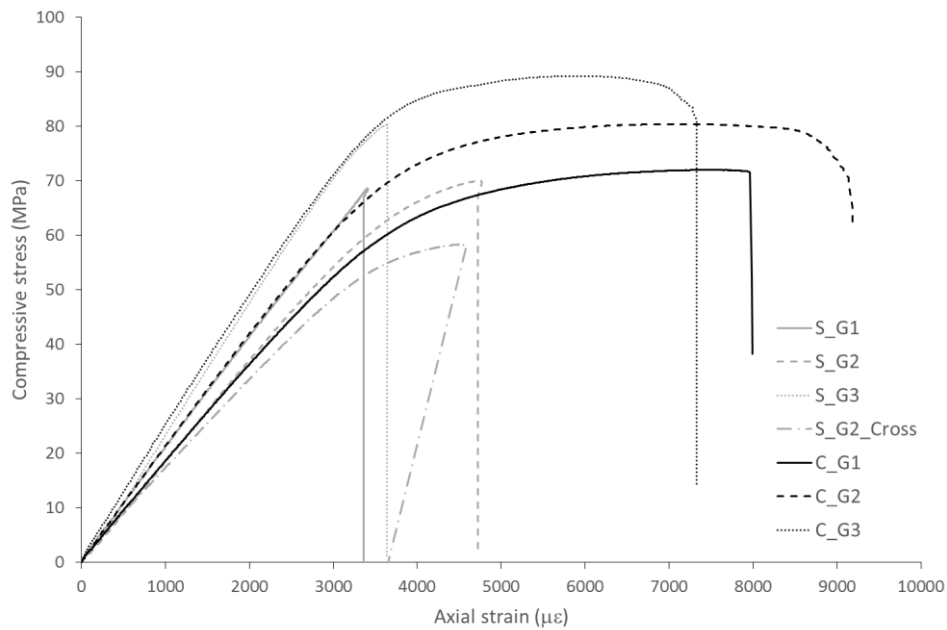
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**Figure 10:** Bending tests, Strain gauge readings for all investigated sections (a) longitudinal strain gauges (SG 1 and SG 3) and (b) transverse strain gauge (SG 2)



**Figure 11:** Shear tests failure modes (a) failure in the timber for all sections but C\_G2\_Cross (shown for S\_G3) and (b) failure in the butt joint for C\_G2\_Cross

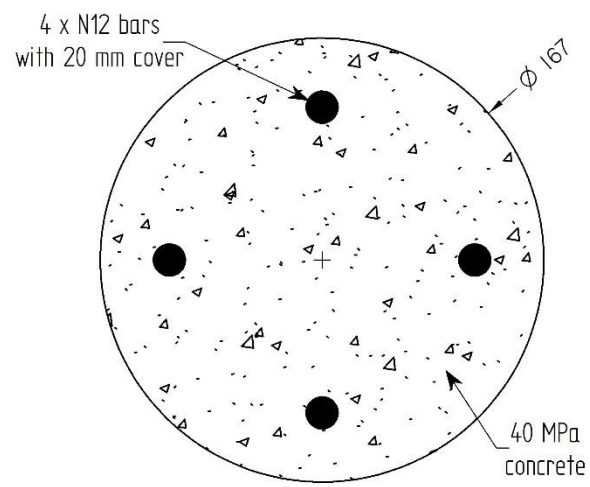


**Figure 12:** Compression tests, Stress-Strain curves ( $\sigma$ - $\epsilon$ ) of all investigated sections





**Figure 13:** Compression tests failure modes, (a) local buckling of the wall for the compact sections (shown for C\_G1) and (b) sudden failure with the sections bursting into strips for the slender sections with no cross-banded veneers (shown for S\_G2)



**Figure 14:** Concrete section used for comparison purposes

**Table 1:** Average compressive and tensile strengths of the material (numbers in brackets indicate the number of tests on which the average and Coefficient of Variation (CoV) are calculated)

Set	Half cross-section 1 <sup>(1)</sup>				Half cross-section 2 <sup>(1)</sup>			
	$\sigma_{comp}$ (MPa)	CoV (%)	$\sigma_{tens}$ (MPa)	CoV (%)	$\sigma_{comp}$ (MPa)	CoV (%)	$\sigma_{tens}$ (MPa)	CoV (%)
S_G1	61.2 (3)	4.2	109.5 (5)	21.4	58.6 (3)	<0.1	96.7 (3)	7.3
S_G2	65.5 (3)	5.3	101.0 (4)	31.1	69.2 (3)	3.5	119.1 (5)	11.4
S_G3	72.7 (3)	3.2	114.0 (5)	31.9	77.9 (3)	3.4	-- <sup>(2)</sup>	-- <sup>(2)</sup>
S_G2_Cross <sup>(3)</sup>	54.0 (3)	0.5	94.4 (5)	5.3	59.9 (2)	12.3	88.9 (5)	15.7
C_G1	67.0 (2)	1.9	99.3 (5)	6.9	64.4 (4)	3.4	96.3 (5)	7.7
C_G2	67.8 (2)	7.2	117.2 (5)	8.7	66.7 (4)	1.6	134.0 (5)	11.8
C_G3	77.9 (2)	3.6	133.0 (5)	8.8	71.3 (3)	2.4	135.8 (5)	11.5

<sup>(1)</sup>: Half cross-section #1 in tension and half cross-section #2 in compression during the bending and shear tests

<sup>(2)</sup>: Samples lost by the external company which CNC cut the samples

<sup>(3)</sup>: Strengths calculated using the gross measured cross-sectional area which includes cross-banded veneers

**Table 2:** Average measured moisture content (MC) for material testing and full cross-sections (numbers in brackets indicate the number of samples on which the average and Coefficient of Variation (CoV) are calculated)

Sample type	Test type	MC (%)	CoV (%)
Material testing	Compression	13.7 (10)	3.4
	Tension	11.3 (14)	5.7
Full cross-sections	Bending	13.7 (4)	4.2
	Shear	12.2 (4)	1.9

**Table 3:** Bending tests results

Set	Capacity $M_b$ (kN.m)	Strength $f_b$ (MPa)	MOE $E_s$ (MPa)	Failure mode
S_G1	20.1	96.9	20154	Compression (buckling) failure
S_G2	21.0	96.3	23252	Tension failure
S_G3	24.6	116.7	27883	Compression (buckling) failure
S_G2_Cross <sup>(1)</sup>	19.3	88.5	14947	Failure in joint between 1/2 cross-sections
C_G1	38.7	116.1	18590	Tension failure
C_G2	39.1	119.0	21666	Failure at support with steel clamps
C_G3	42.6	128.6	23331	Failure at support with steel clamps

<sup>(1)</sup>: Strength  $f_b$  calculated using the gross measured cross-section which includes cross-banded veneers

**Table 4:** Shear tests results

Set	Capacity $V_s$ (kN)	Strength $f_s$ (MPa)
S_G1	30.5	10.3
S_G2	29.5	9.5
S_G3	30.9	9.9
S_G2_Cross <sup>(1)</sup>	32.0	10.4
C_G1	53.9	9.5
C_G2	60.8	10.7
C_G3	58.5	10.6

<sup>(1)</sup>: Strength  $f_s$  calculated using the gross measured cross-section which includes cross-banded veneers

**Table 5:** Compression tests results

Set	Capacity $R_c$ (kN)	Strength $f_c$ (MPa)	MOE $E_s$ (MPa)	Section strength $f_c$ / material strength $\sigma_{comp}$	$E_s$ (bending) / $E_s$ (compression)
S_G1	438.2	68.6	18824	1.15	1.07
S_G2	451.4	70.0	20592	1.04	1.13
S_G3	488.0	80.3	24709	1.07	1.13
S_G2_Cross <sup>(1)</sup>	372.3	58.4	16343	1.02	0.91
C_G1	784.0	72.0	17852	1.10	1.04
C_G2	897.8	80.4	20849	1.20	1.04
C_G3	992.4	89.3	24529	1.20	0.95

<sup>(1)</sup>: Strength  $f_c$  calculated using the gross measured cross-section which includes cross-banded veneers

**Table 6:** Structural efficiency of circular timber, steel and reinforced concrete sections

Section	Bending			Shear		Compression		
	$M_b$ (kN.m)	$M_b$ / linear weight (kN.m/kg)	Stiffness $EI$ (kN.m <sup>2</sup> )	$V_s$ (kN)	$V_s$ / linear weight (kN/kg)	$R_c$ (kN)	$R_c$ / linear weight (kN/kg)	Stiff- ness $EA$ (kN)
Timber (C_G2)	39.1	4.4	$6.15 \times 10^2$	60.8	6.8	897.8	100.0	$2.37 \times 10^5$
Steel (168×4.8)	44.8	2.3	$1.65 \times 10^3$	311.2	16.1	864.5	44.6	$4.94 \times 10^5$
Concrete	11.9	0.2	$1.22 \times 10^3$	92.0	1.8	946.4	18.0	$7.18 \times 10^5$